

PORTLAND CEMENT ASSOCIATION
RESEARCH AND DEVELOPMENT LABORATORIES

Development Department

•

Bulletin D24

PRECAST CONCRETE GIRDERS
REINFORCED WITH
HIGH STRENGTH DEFORMED BARS

By J. R. Gaston and Eivind Hognestad

Authorized Reprint from Copyrighted
Journal of the American Concrete Institute
Oct. 1958, Proceedings Vol. 55, p. 469

**Bulletins Published by the
Development Department
Research and Development Laboratories
of the
Portland Cement Association**

- D1 —“Influence of Soil Volume Change and Vegetation on Highway Engineering,”** by E. J. FELT.

Reprinted from *Twenty-Sixth Annual Highway Conference of the University of Colorado*, May, 1953.

- D2 —“Nature of Bond in Pre-Tensioned Prestressed Concrete,”** by JACK R. JANNEY.

Reprinted from *Journal of the American Concrete Institute* (May, 1954); *Proceedings*, 50, 717 (1954).

- D2A—Discussion of the paper “Nature of Bond in Pre-Tensioned Prestressed Concrete,”** by P. W. ABELES, K. HAJNAL-KONYI, N. W. HANSON and Author, JACK R. JANNEY.

Reprinted from *Journal of the American Concrete Institute* (December, Part 2, 1954); *Proceedings*, 50, 736-1 (1954).

- D3 —“Investigation of the Moisture-Volume Stability of Concrete Masonry Units,”** by JOSEPH J. SHIDELER.

Published by Portland Cement Association, Research and Development Laboratories, Chicago, Illinois, March 1955.

- D4 —“A Method for Determining the Moisture Condition of Hardened Concrete in Terms of Relative Humidity,”** by CARL A. MENZEL.

Reprinted from *Proceedings, American Society For Testing Materials*, 55 (1955).

- D5 —“Factors Influencing Physical Properties of Soil-Cement Mixtures,”** by EARL J. FELT.

Reprinted from *Bulletin 108 of the Highway Research Board*, p. 138 (1955).

- D6 —“Concrete Stress Distribution in Ultimate Strength Design,”** by E. HOGNESTAD, N. W. HANSON and D. MCHENRY.

Reprinted from *Journal of the American Concrete Institute* (December, 1955); *Proceedings*, 52, 455 (1956).

- D6A—Discussion of the paper, “Concrete Stress Distribution in Ultimate Strength Design,”** by P. W. ABELES, A. J. ASHDOWN, A. L. L. BAKER, ULF BJUGGREN, HENRY J. COWAN, HOMER M. HADLEY, KONRAD HRUBAN, J. M. PRENTIS, E. ROSENBLUETH, G. M. SMITH, L. E. YOUNG, A. J. TAYLOR, and Authors, E. HOGNESTAD, N. W. HANSON, and D. MCHENRY.

Reprinted from *Journal of the American Concrete Institute* (December, Part 2, 1956); *Proceedings* 52, 1305 (1956).

—Continued on Inside Back Cover



research and development

Title No. 55-31

PRECAST CONCRETE GIRDERS reinforced with HIGH STRENGTH DEFORMED BARS*

By J. R. GASTON and EIVIND HOGNESTAD†

Two 0.38-scale model roof girders were tested to develop an unusual type of precast concrete building frame. The roof girder selected departs from customary practice primarily by its slender and graceful T cross section, by its high strength longitudinal tension reinforcement, and by its inclined stirrup reinforcement. Structural design was based on the ultimate strength design procedure given in the appendix of the 1956 ACI Building Code, with some departures justified by the model girder test results. Twenty 58-ft girders were later manufactured for two laboratory buildings.

UNUSUAL PRECAST BUILDING FRAME

In early design stages of two new laboratory buildings at the Portland Cement Association Laboratories, it was decided that recent developments in structural concrete would be utilized to a greater extent than is at present customary in building construction. Functional requirements of the two buildings, the Structural Laboratory and the Fire Research Center, called for an open crane-bay superstructure about 58 ft wide and 40 ft high from first floor to roof level. Furthermore, special requirements related to the use of the buildings as laboratories called for extremely heavy box-like foundations in both cases.

The superstructures were ideal for precast concrete construction since considerable repetition of identical structural units was possible within each

*Presented at the ACI 54th annual convention, Chicago, Ill., Feb. 26, 1958. Title No. 55-31 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 4, Oct. 1958, *Proceedings* V. 55. Separate prints are available at 60 cents each. **Discussion** (copies in *indicate*) should reach the Institute not later than Jan. 1, 1959. Address P. O. Box 4734, Redford Station, Detroit 19, Mich.

†Members American Concrete Institute, Associate Development Engineer and Manager Structural Development Section, respectively, Portland Cement Association, Chicago, Ill.

structure as well as between the two structures. Preliminary designs therefore turned to a system of precast columns, wall panels, roof girders, roof slabs, spandrel beams, and crane girders.

The ultimate strength design method given in the appendix of the 1956 ACI Building Code (ACI 318-56)* was used in the structural design. However, it was decided to exceed moderately the 60,000 psi maximum value for stress in reinforcement at ultimate strength given by Section A603(e). Design of some details, such as roof girder web reinforcement, also departed somewhat from the limits and principles of ACI 318-56.

Thus, the experimental investigation reported herein was undertaken to develop and to test an unusual type of precast concrete building frame. The roof girder selected departs from customary practice by its slender T-shaped cross section, by its high strength longitudinal reinforcement, and by its inclined stirrup reinforcement.

Two model girders were tested. The purpose of the first test was to determine the general behavior of the girder from zero load to ultimate strength, with particular attention given to flexural cracking and performance at service load. In the second girder test, attention was focused on adequacy of web reinforcement, action of face bars provided to reduce the width of cracks in the web, and the strength and performance of the girder-column connection.

TEST GIRDERS

Prototype frames

The frame system and the roof girder shape were selected as a result of suggestions originating with Ulf Bjuggren and Torsten Lundin of Stockholm, Sweden. Columns 2 ft square and about 38 ft tall are fixed to the foundation. Crane girders and spandrel beams span 22 ft longitudinally between columns, and roof girders span 58 ft transversely between pairs of columns. The roof girders are connected to the columns in a manner designed to permit rotation without development of negative frame corner moments. However, the connection permits a horizontal force to be transmitted in each frame through the girder from one column top to another. Thereby both columns of a frame may be designed to "share" sidesway moments due to wind and crane loads. The maximum compression so transmitted through the girder is less than 5 percent of the internal compression due to bending of the girder.

The roof girders selected have a T-shaped cross section 4 ft deep at mid-span and 3 ft deep at the ends. Structural frame design was carried out for a design minimum concrete strength of 5000 psi and a design minimum yield point for the main longitudinal tension reinforcement of 70,000 psi. A longitudinal tension reinforcement of eight #9 bars with a total area of 8.0 sq in. satisfied design requirements.

*ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-56)," ACI JOURNAL, May 1956, *Proc.* V. 52, pp. 913-986. Further reference to Code is given by section only.

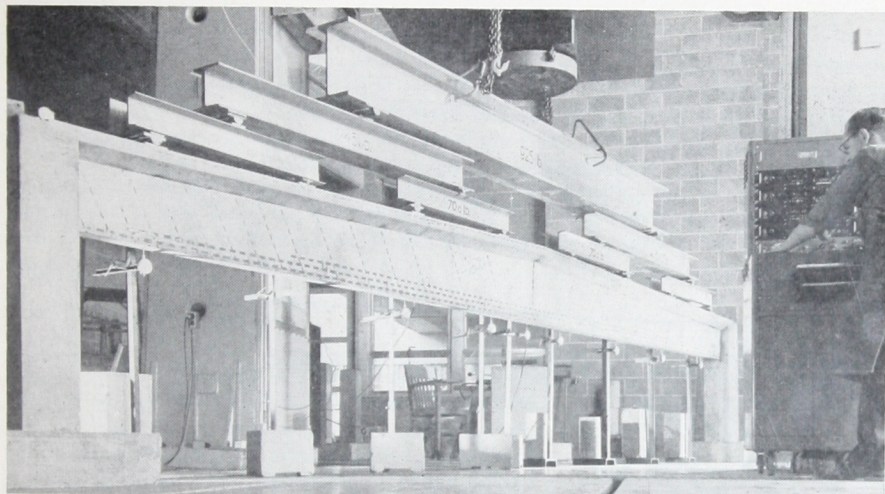


Fig. 1—Testing arrangement for Girder 1

Test specimens

A 0.38-scale model of the prototype was the largest specimen that could be conveniently handled in the laboratory. Fig. 1 and 2 show over-all views of the testing arrangement for the two test girders, and their dimensions are given in Fig. 3. The total depth of the test girders was $0.38 \times 4 \text{ ft} = 18\frac{1}{4} \text{ in.}$ at midspan, and $0.38 \times 3 \text{ ft} = 13\frac{1}{2} \text{ in.}$ at the ends. To simulate the roof loading of the prototype, the test girders were loaded at eight points as shown in Fig. 3.

The columns of the prototype frames were designed in accord with the appendix and other applicable provisions of ACI 318-56. The column design procedures involved, therefore, are substantiated by numerous previous tests. It was necessary in this investigation, however, to develop a satisfactory connection between girders and columns. To accomplish

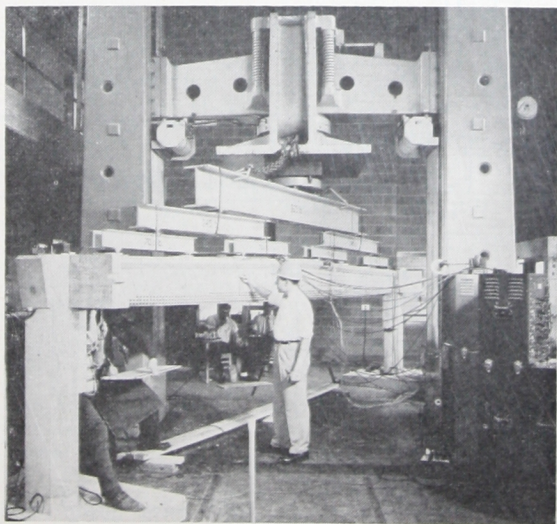


Fig. 2—Testing arrangement for Girder 2

The physical properties of these bars are given in Fig. 5. The compression reinforcement, stirrups, and face bars of the test girders were #2 deformed bars having an area of 0.045 sq in. and a yield point of 55,000 psi.

The girders were cast in plastic coated plywood forms with close adherence to the dimensions given in Fig. 3. The fabrication and placing of reinforcement cages was carried out to tolerances of less than $\frac{1}{4}$ in. Concrete was placed in the test girders and companion cylinders by internal vibration. Following 3 days of moist curing, all specimens were stored dry in the laboratory until they were tested. It was noted that the taper of the web permitted the girders to be lifted out of the forms without dismantling the forms. This could simplify form design and mass production of the type of girder involved.

Design of test girders

The main tension reinforcement of Girder 1 consisted of six #4 bars with a total area of 1.20 sq in. This represents closely the area corresponding to the prototype, namely, $8 \times 0.38^2 = 1.16$ sq in. To design web reinforcement for the test girders, and to study their service load behavior, it was necessary to calculate a service load for the test girders corresponding to that of the prototype. For this purpose, compression reinforcement and the small axial force were neglected and the prototype's 70,000 psi yield point and 5000 psi concrete strength were assumed. Flexural strength is controlled by the section at the load points nearest midspan, for which the effective depth $d = 15.5$ in. By ACI 318-56, Section A607(a), the depth to the neutral axis at ultimate strength $= 1.30 qd = 1.30 \times 0.12 \times 15.5 = 2.4$ in. This depth is less than the average flange depth of $t = 3.0$ in., and $q = 0.12$ is less than the limiting value at 0.40 given by Section A605(c), so that the ultimate moment is given by Section A605(b):

$$M_u = bd^2 f_c' q (1 - 0.59q) = 1210 \text{ kip-in.} \dots \dots \dots (1)$$

in which

$$t = \frac{1}{9} \left[3.5^2 + \frac{9 - 3.5}{2} (2 + 3.5) \right] = 3 \text{ in.}$$

f_c'	= cylinder strength = 5000 psi
q	= $A_s f_y / bd f_c' = 0.12$
A_s	= area of tension reinforcement = 1.2 sq in.
f_y	= tension steel yield point = 70,000 psi

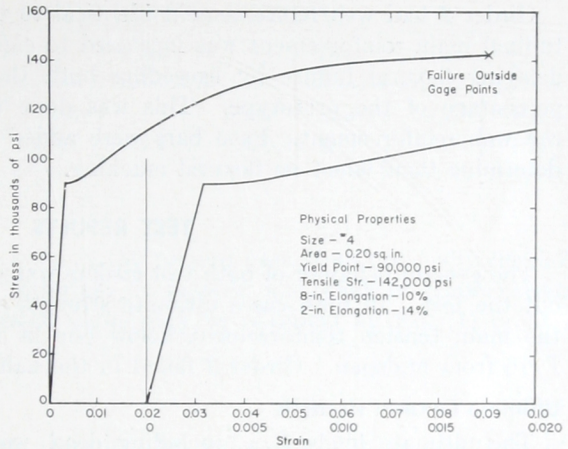
M_u = ultimate moment
 b = total flange width = 9 in.
 d = effective depth = 15.5 in.

For the test girder loading, the ultimate load is:

$$P_u = M_u \frac{8}{L} = 1210 \frac{8}{266} = 36.4 \text{ kips}$$

The live load of the prototype is less than the dead load, so that a load factor $K = 1.8$ applies. Hence, the test girder service load including the weight of the girder is $36.4/1.8 = 20.2$ kips, and the service load shear, V , at each beam end is 10.1 kips.

Fig. 5—Physical properties of
AISI No. 9261 deformed bars



Shearing stress at the girder end is:

$$v = \frac{V}{bjd} = \frac{10,100}{3 \times 0.9 \times 12.75} = 293 \text{ psi} \dots \dots \dots (2)$$

which exceeds the 240-psi ceiling given by Section 803(c) for beams with vertical stirrups used alone as web reinforcement. Stirrups inclined 60 deg were chosen as web reinforcement for the prototype. Since high strength longitudinal reinforcement cannot be welded in the field, the stirrups were only tied to the main reinforcement with soft wire, although Section 802(a) calls for inclined stirrups to be "welded or otherwise rigidly attached to the longitudinal steel."

The stirrups were designed by Section 804(d) of the ACI Code:

$$A_v = \frac{V' s}{f_v j d (\sin \alpha + \cos \alpha)} \dots \dots \dots (3)$$

in which

- | | |
|---|--------------------------------|
| A_v = stirrup area = $2 \times 0.045 = 0.09$ sq in. | $jd = 0.9d$ |
| s = horizontal stirrup spacing | α = stirrup inclination |
| f_v = tensile unit stress = 20,000 psi | = 60 deg |

By Section 801(d), the shear, V' , carried at service load by web reinforcement should be taken as the excess over a 90 psi shearing stress permitted for the concrete of an unreinforced web. It was felt that, in the present case of an unusually slender girder with high strength materials, the resulting stirrup reinforcement would be too light in the midspan region. It was chosen, therefore, to design the stirrups throughout the girder span for a value of V' equal to 2/3 of the total shear present at a section. The stirrup reinforcement designed in this manner is shown in Fig. 3.

Girder 2 had web reinforcement identical to that of Girder 1. The longitudinal main reinforcement was increased to eight #4 bars, thus still further delaying flexural failure by exceeding both the yield point and the steel percentage of the prototype. This was done to ascertain the strength of the web reinforcement. Face bars were added to one-half of the girder to determine their effect on flexural cracking.

TEST RESULTS

The general behavior of both test girders was entirely satisfactory throughout the test. In both cases ultimate strength was governed by yielding of the main tension reinforcement below one of the two load points located $L/16$ from midspan. Girder 2 failed in the half without face bars.

Ultimate flexural strength

The ultimate loads P_{test} , including dead weight of girder and loading frame, are given in Table 1 together with other pertinent strength data. Using 90,000 psi longitudinal reinforcement, the average flange depth of 3.0 in. is less than the depth to the neutral axis given by $1.30 qd$ [Section A607(a)]. Accordingly, the ultimate moment is given by Section A607(b) of the ACI Code:

$$M_u = (A_s - A_{sf}) f_y d [1 - 0.59 (q_w - q_f)] + A_{sf} f_y (d - 0.5t) \dots \dots (4)$$

in which for Girders 1 and 2, respectively:

A_s = area of tension reinforcement = 1.20 and 1.60 sq in.	b' = average width of web = 3 in.
$A_{sf} = 0.85 (b - b') t f'_c / f_y = 0.804$ and 0.890 sq in.	t = average flange thickness = 3 in.
f_y = yield point of tension reinforcement = 90,000 psi	f'_c = cylinder strength = 4730 and 5230 psi
b = flange width = 9 in.	d = effective depth = 15.5 and 15.25 in.
	$q_w = A_s f_y / b' d f'_c = 0.491$ and 0.602
	$q_f = A_{sf} f_y / b' d f'_c = 0.329$ and 0.336

The corresponding calculated ultimate loads, P_{calc} , are given in Table 1. It may be noted that the 60,000 psi yield point limit given by Section A603(e) was exceeded and that compression reinforcement was neglected. The values of $q_w - q_f$ given in Table 1 are substantially below the maximum value of 0.40 given by Section A607(c), so that Eq. (4) is applicable.

Ultimate flexural strength was also calculated by more refined methods using a curved stress-strain relationship for the concrete and using the actual girder cross section. For an assumed trapezoidal steel stress-strain relationship, calculated ultimate loads were obtained within a few percent of those given in Table 1 for Eq. (4). Using the actual steel stress-strain curve of Fig. 5, however, calculations give a steel strain of about 0.01 at ultimate strength, which corresponds to a steel stress by strain hardening of 100,000 psi. The resulting calculated ultimate loads are in close agreement with the observed ultimate loads, P_{test} , given in Table 1.

TABLE 1—GIRDER FLEXURAL STRENGTH

Girder No.	f'_c , psi	A_s , sq in.	d ,* in.	q †	$1.3qd$, in.	$q_w - q_f$	P_{test} , kips	P_{calc} , kips	$\frac{P_{test}}{P_{calc}}$
1	4730	1.20	15.5	0.164	3.3	0.162	50.4	45.4	1.11
2	5230	1.60	15.25	0.301	4.0	0.266	67.2	57.6	1.16

*At load points $L/16$ from midspan.

†Based on full 9-in. flange width.

It may be concluded that, even in the present case of an unusually slender girder with high strength materials, a reliable and slightly conservative ultimate flexural strength design value may be obtained by Eq. (1) or (4), whichever is applicable.

Girder deflection

Deflections of Girder 1 were measured by dial gages (Fig. 1), and those of Girder 2 were obtained utilizing scales fastened to the girder and read with a surveyor's level. Midspan deflections are given in Fig. 6a as a function of the total load. It is seen that Girder 1, which had tension reinforcement corresponding to that of the prototype, had a deflection of about 1 in. at full service load. This corresponds to $1/0.38 = 2.6$ in. for the prototype. The prototype service load consists of about 27 percent live load and 73 percent dead load. Assuming that dead load deflections will be doubled by the effect of creep, the total prototype deflection is $2.6 (2 \times 0.73 + 0.27) = 4.5$ in. The prototype girders were therefore built with a camber of 4.5 in.

Girder deflection was calculated using the moment of inertia of the gross concrete section, considering the effect of the variable girder depth, neglecting reinforcement, and using a modulus of elasticity equal to $1000 f'_c$. A load-deflection line obtained for 5000-psi concrete strength is plotted in Fig. 6a. It is seen that, for the unusual girder involved, the measured deflection at service loads exceeds the calculated value.

By a second method, deflection was calculated on the basis of the following assumptions: (1) concrete and steel stress are proportional to strain; (2) the concrete section is cracked, and no tensile stress is carried by the concrete below the neutral axis; and (3) the modular ratio is $n = 30,000/f'_c$ as given by Section 305. The load-deflection curves obtained by this cracked-section method are also plotted in Fig. 6a. At low loads, the actual deflections were less than those so computed, because the girders were actually not cracked. At loads near the yield loads, actual deflections exceeded those calculated, because concrete and steel were approaching plastic action. In the region of service loads, however, deflection computed by the cracked-section method is in close agreement with measured deflections.

Flexural strain

Strain measurements were made on Girder 2 only. Flexural strain was measured near the two central load points at sections $22\frac{5}{8}$ in. from midspan.

At each section, strain in the compression flange was measured by five 6-in. electric strain gages attached to the concrete surface. A 10-in. Whittemore mechanical gage was used to measure strain at four levels in the web. Finally, strains for all four steel layers were observed by electric gages attached to the reinforcement, waterproofed, and embedded in the concrete.

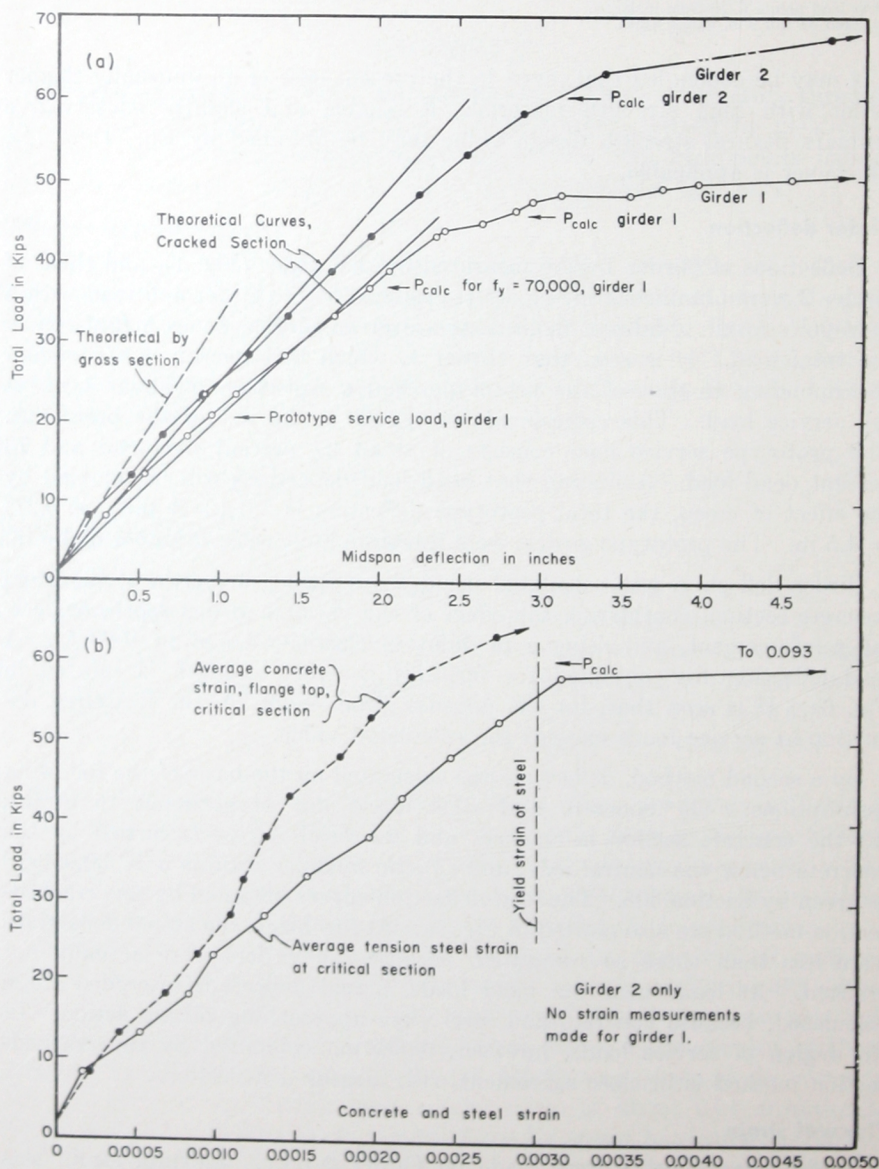


Fig. 6—Deflection and strain data

The distribution of strain across both sections was linear from the first load increment to failure. This substantiates numerous previous observations justifying the design assumption that plane sections normal to the axis remain plane after bending. Average concrete and steel strain are given in Fig. 6b as a function of load. The concrete strain at failure exceeded 0.003. The steel strain exceeded 0.01, confirming that the excess ultimate flexural strength over calculated values was due to strain hardening of the reinforcement.

Control of cracking

Reinforced concrete must be expected to crack at loads considerably below the full design service load. For a modular ratio $n = 30,000/f'_c$, tensile stress in the concrete at the tension steel level is $f_t = f_s f'_c / 30,000$. Hence, if the modulus of rupture of the concrete is $f_t = 0.15 f'_c$, cracking must be expected at a tensile steel stress $f_s = 4500$ psi. It is known that, for a given beam, the width of cracks is essentially proportional to steel stress, and it has been customary to limit crack width by limiting the allowable steel stress to 20,000 psi even when beam strength requirements would permit a higher steel stress.

The high strength reinforcement selected for the prototype girders has a minimum yield point of 70,000 psi. For a load factor of 1.8, the tension steel stress at service load is then 40,000 psi. Accordingly, it was necessary to develop other means of crack control than that of limiting the steel stress. The slender T-shaped cross section of the girders was chosen to reduce costs and dead weight and to realize a graceful architectural appearance. In addition, it was an important function of the slender section to obtain a crack pattern consisting of numerous very fine cracks rather than a few wide cracks.

The width of cracks was measured with a graduated microscope in both girder tests, and the location and extension of cracks were noted at several load levels. The crack patterns at loads near that corresponding to the prototype service load (20.2 kips) are given in Fig. 7.

The width of flexural cracks in the constant moment region at midspan at a level 0.75 in. above the bottom of the girders is given in Fig. 7 as a function of steel stress. Both the maximum crack width and the average width are given. Cracks near the beam ends, including diagonal cracks, were always narrower than those near midspan. Comparison is made with the results of an auxiliary group of tests involving three 8 x 16-in. rectangular beams loaded at third-points of an 11-ft span.

It is seen in Fig. 7 that a change from rectangular to T-shaped cross section has a pronounced effect on both the maximum and the average crack width. At the prototype service load steel stress of 40,000 psi, the average crack width for the test girders is 0.001 to 0.002 in., which is the crack width for the rectangular beams at the customary steel stress of 20,000 psi. To control

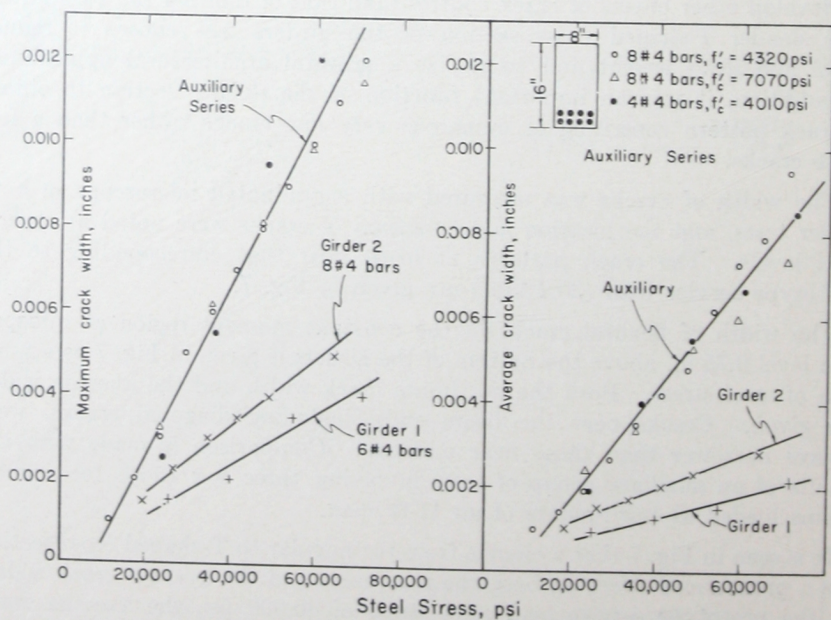
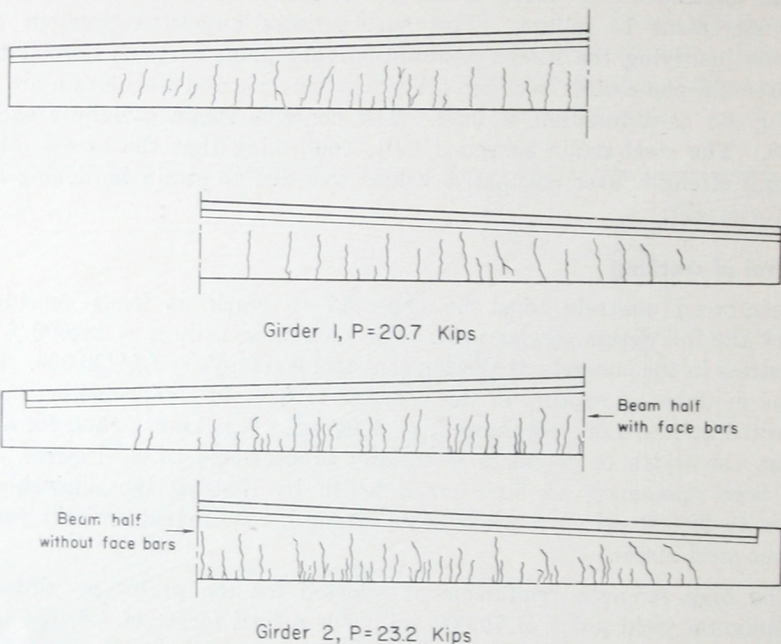
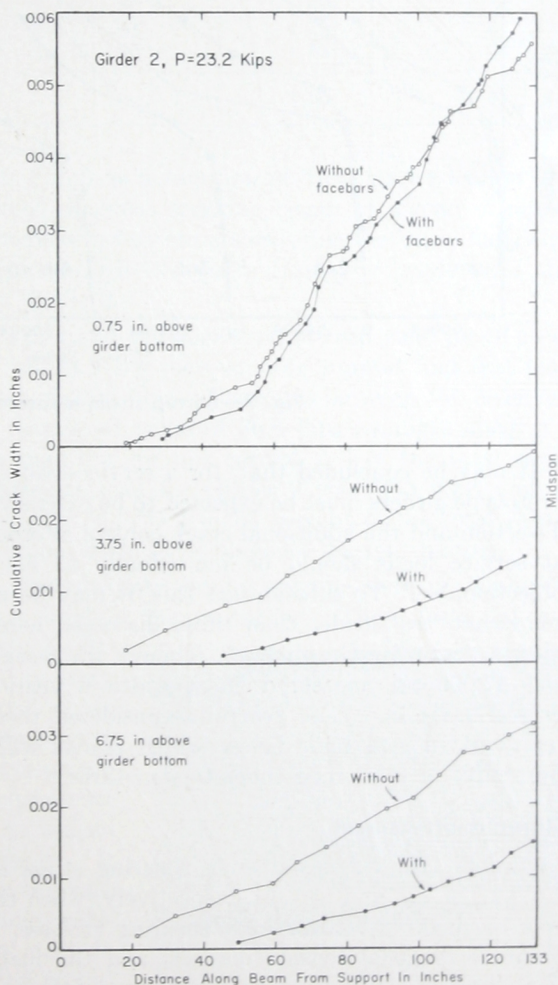


Fig. 7—Crack studies

cracking, therefore, the selection of a narrow girder width is an effective alternate in design to the customary measure of limiting steel stress.

To determine their effect on cracking, face bars were provided only in one half of Girder 2. Fig. 8 gives crack width cumulated for all cracks from

Fig. 8—Crack control by face bars



the girder end to midspan. At a level 0.75 in. above the bottom of the girder, the curve for the girder half with face bars practically coincides with that for the half without face bars. At the higher levels, however, Fig. 8 shows that the face bars reduced the cumulative crack width substantially. The face bars effectively control cracking in the beam web, and they were used in the entire span of the prototype girders.

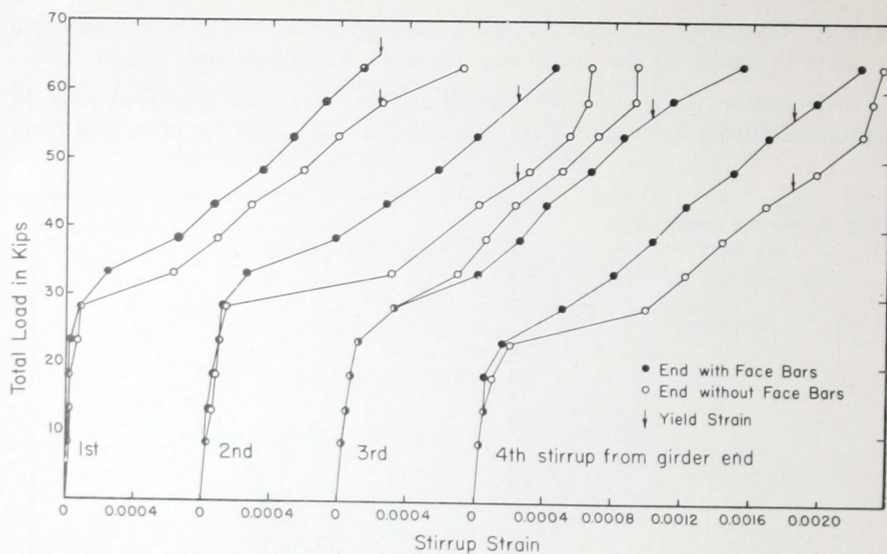


Fig. 9—Stirrup strain measurements

It may be concluded that, for a service-load steel stress of 40,000 psi, the prototype girders must be expected to be extensively cracked. By the slender T-section and the additional crack control afforded by face bars, these cracks at service loads should be individually so narrow that they will not be objectionable. To substantiate this, it may be noted that 76-ft roof girders somewhat less slender than those discussed herein were built in Stockholm after a design by Granholm.* Though the design steel stress at service load was 47,000 psi, measured crack width 6 years after construction did not exceed 0.004 in. It is generally considered that a crack width of 0.01 in. represents a safe limit below which cracks will not impair the beauty or durability of reinforced concrete structures.

Stirrup reinforcement

Girders 1 and 2 sustained a shearing stress at their ends, calculated by Eq. (2), of 723 and 980 psi, respectively, when the ultimate flexural strength was reached. At these high shearing stresses, there was no indication of shear or diagonal tension distress, and the inclined stirrups aided by face bars limited diagonal crack width to a few thousandths of an inch. For the prototype girders, therefore, a 300 psi design shearing stress at service load is entirely satisfactory, in spite of the fact that the 240 psi maximum value given by Section 803(c) is exceeded.

Electric strain gages were attached to four stirrups at each end of Girder 2 at middepth of the web. The gages were waterproofed and embedded.

*Granholm, H., "Modern Developments in Reinforced Concrete Design and the Influence of Research," *Proceedings, Building Research Congress, London, 1951, Division 1*, pp. 51-56.

TABLE 2—YIELD LOADS FOR END STIRRUPS

Stirrup No. from girder end	P_{yield} , kips		$P_{yield}/(36.6 \text{ kips})$	
	With face bars	Without face bars	With face bars	Without face bars
1st	65	58	1.78	1.58
2nd	58	47	1.58	1.28
3rd	56	61	1.53	1.67
4th	56	46	1.53	1.26
Average			1.60	1.45

The strain data are given in Fig. 9 as a function of total girder load. The stirrup strains were small until diagonal cracking began at a load of about 20 kips. The strains then increased, but more slowly for the beam half with face bars. The girder loads at which the yield strain was reached for the various stirrups are given in Table 2.

For the 4.25-in. stirrup spacing and the 55,000 psi stirrup yield point used, Eq. (3) gives an average V' of 18.3 kips between the support and first load point, and a total girder load of 36.6 kips. The ratio between the observed stirrup yield load and this calculated value of 36.6 kips averages about 1.5 in Table 2. This indicates that the stirrups carried 2/3 of the total shear when they yielded. The ultimate load for Girder 2 was 67.2 kips, and at this load the stirrups carried only $36.6/67.2 = 55$ percent of the total shear. Even so, there was no indication of shear or diagonal tension distress.

To facilitate fabrication, intermediate grade stirrups with a minimum yield point of 40,000 psi are used in the prototype. These tests indicate that such stirrups may safely be designed by Section 804(d), but to carry 2/3 of the total service load shear throughout the girder span at a stirrup unit stress of 20,000 psi. The ultimate flexural strength of a prototype girder so designed can be fully developed without distress due to shear. It seems entirely satisfactory in this case to use inclined stirrups without welding them to the longitudinal steel, in spite of the fact that Section 802(a) calls for welding.

On the other hand, since no shear distress was observed in either of the two girder tests, it is entirely possible that stirrups designed to carry only the excess over a 90 psi shearing stress permitted on an unreinforced web

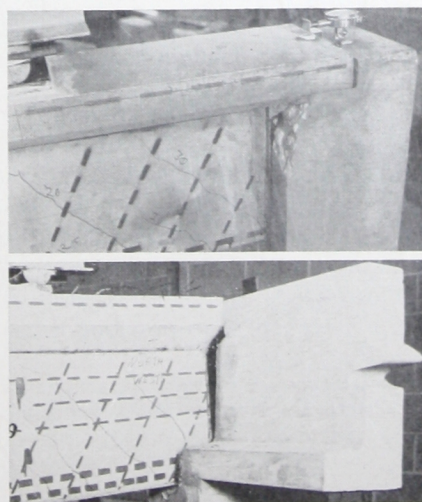


Fig. 10—Girder-column connection after test. Top—Girder 1; bottom—Girder 2

could suffice. It is the opinion of the writers, however, that any major roof girder, any beam with high strength reinforcement, and any beam with an unusually thin web should be provided with some web reinforcement throughout its span. This is particularly so for precast members because unforeseen erection loads, such as torsion, could seriously damage an unreinforced web.

Girder-column connections

Both girders were tested to failure in flexure with no indication of structural distress in the girder-column connection. Even the large rotations after the girders reached the plastic stage of large deflection were accommodated adequately. As shown in Fig. 10, bearing of the flange of Girder 1 caused local spalling of the column corner. This took place at service load, and could have been prevented by caulking rather than grouting the space between the flange and the column. For Girder 2 and for the prototype, however, the girder flange was discontinued at the face of the spandrel beams as seen in the lower photograph of Fig. 10. The performance of this latter connection was excellent in all respects.

CONCLUSIONS

Since the 1956 ACI Building Code reflects the state of reinforced concrete design as represented by the best practice of the day, it was considered that any departure from the Code provisions should be well substantiated.

On the basis of tests of two 0.38-scale model girders, it was found entirely safe and satisfactory in the present particular case to depart in design criteria from the Code provisions as follows:

- (1) The 60,000 psi maximum value for stress in reinforcement at ultimate strength as given by Section A603(e) was increased to 70,000 psi.
- (2) The 240 psi maximum value given by Section 803(c) for unit shearing stress at service load of beams with stirrup reinforcement only was increased to 300 psi.
- (3) The provision of Section 802(a) that inclined stirrups must be "welded or otherwise rigidly attached to the longitudinal steel" was waived.
- (4) It was considered desirable in the present case to use a more conservative stirrup design procedure than that given by Sections 801 and 802. Two-thirds of the total shear was considered carried by stirrups throughout the girder span, even where the shearing stress "permitted on the concrete of an unreinforced web" was not exceeded.
- (5) Face bars were provided to control cracking in the girder web.

Discussion of this paper should reach ACI headquarters in triplicate by Jan. 1, 1959, for publication in the June 1959 JOURNAL.

- D7 —“Ultimate Flexural Strength of Prestressed and Conventionally Reinforced Concrete Beams,”** by J. R. JANNEY, E. HOGNESTAD and D. McHENRY.

Reprinted from *Journal of the American Concrete Institute* (February, 1956); *Proceedings*, 52, 601 (1956).

- D8 —“Resurfacing and Patching Concrete Pavement with Bonded Concrete,”** by EARL J. FELT.

Reprinted from *Proceedings of the Highway Research Board*, 35 (1956).

- D9 —“Review of Data on Effect of Speed in Mechanical Testing of Concrete,”** by DOUGLAS McHENRY and J. J. SHIDELER.

Reprinted from *Special Technical Publication No. 186*, published by American Society For Testing Materials (1956).

- D10—“Laboratory Investigation of Rigid Frame Failure,”** by R. C. ELSTNER and E. HOGNESTAD.

Reprinted from *Journal of the American Concrete Institute* (January, 1957); *Proceedings* 53, 601 (1957).

- D11—“Tests to Evaluate Concrete Pavement Subbases,”** by L. D. CHILDS, B. E. COLLEY and J. W. KAPERNICK.

Reprinted from the *Journal of the Highway Division of The American Society of Civil Engineers*, Paper No. 1297, Vol. 83, HW3, July, 1957.

- D12—“Ultimate Strength of Reinforced Concrete in American Design Practice,”** by E. HOGNESTAD.

Reprinted from *Proceedings of a Symposium on the Strength of Concrete Structures*, London May, 1956.

- D13—“Effect of Variations in Curing and Drying on the Physical Properties of Concrete Masonry Units,”** by WILLIAM H. KUENNING and C. C. CARLSON.

Published by Portland Cement Association, Research and Development Laboratories, Chicago, Illinois, November, 1956.

- D14—“Lightweight Aggregates for Concrete Masonry Units,”** by C. C. CARLSON.

Reprinted from *Journal of the American Concrete Institute* (November, 1956); *Proceedings*, 53, 491 (1956).

- D15—“Confirmation of Inelastic Stress Distribution in Concrete,”** by EIVIND HOGNESTAD.

Reprinted from the *Journal of the Structural Division, Proceedings of the American Society of Civil Engineers*, Proc. Paper 1189, Vol. 83, No. ST 2, March 1957.

- D16—“Strength and Elastic Properties of Compacted Soil-Cement Mixtures,”** by EARL J. FELT and MELVIN S. ABRAMS.

Reprinted from *Special Technical Publication No. 206*, published by the American Society for Testing Materials (1957).

—Continued on Outside Back Cover

D17—"Lightweight Aggregate Concrete for Structural Use," by J. J. SHIDELER.

Reprinted from *Journal of the American Concrete Institute* (October, 1957); *Proceedings*, **54**, 299 (1957).

D18—"Stresses in Centrally Loaded Deep Beams," by P. H. KAAR.

Reprinted from *Proceedings of the Society for Experimental Stress Analysis*, Vol. 15, No. 1. 77 (1957).

D19—"Strength of Concrete Under Combined Tensile and Compressive Stresses," by DOUGLAS MCHENRY and JOSEPH KARNI.

Reprinted from *Journal of the American Concrete Institute* (April, 1958); *Proceedings*, **54**, 829 (1958).

D20—"General Considerations of Cracking in Concrete Masonry Walls and Means for Minimizing It," by CARL A. MENZEL.

Published by Portland Cement Association, Research and Development Laboratories, Chicago, Illinois, September, 1958.

D21—"Tests of Concrete Pavement Slabs on Open Graded Granular Subbases," by L. D. CHILDS and J. W. KAPERNICK.

Reprinted from the *Journal of the Highway Division, Proceedings of the American Society of Civil Engineers*, Proc. Paper 1800, Vol. 84, HW3, October, 1958.

D22—"Shear Strength of Lightweight Reinforced Concrete Beams," by J. A. HANSON.

Reprinted from *Journal of the American Concrete Institute* (September, 1958); *Proceedings*, **55**, 387 (1958).

D23—"Performance of Subbases Under Repetitive Loading," by B. E. COLLEY and J. NOWLEN.

Reprinted from Bulletin 207 of the *Highway Research Board*, 1958.

D24—"Precast Concrete Girders Reinforced with High Strength Deformed Bars," by J. R. GASTON and EIVIND HOGNESTAD

Reprinted from *Journal of the American Concrete Institute* (October, 1958); *Proceedings*, **55**, 469 (1958).